Blast Damage Mitigation Using Reinforced Concrete Panels and Energy Absorbing Connectors

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Prepared for:
Department of Defense Explosive Safety Board
1996 Explosives Safety Seminar

Abstract

This paper summarizes dynamic structural analysis and physical testing performed in order to develop energy absorbing connectors used for supporting pre-cast reinforced concrete panels subjected to blast waves. The work was conducted under the sponsorship of the Hoechst Celanese Chemical Group, Ltd., (Hoechst Celanese Corporation) who has authorized the release of this information. Reinforced concrete pre-cast panels can be used in new construction or as part of a building upgrade in situations where blast resistance is desired. Traditionally, these panels are mounted with fairly rigid supports and can be designed according to methods in TM 5-1300. The study described in this paper investigated the use of deformable, energy absorbing supports to achieve a reduction in panel damage and to reduce loads transferred to supporting frames. A specific energy absorbing mount was developed and tested under blast conditions. This paper includes a description of important non-dimensional parameters relating blast loading to performance, application in design, and a summary of the blast testing conducted. The shock absorbing systems (panels plus mounts) were tested at quarter-scale, half-scale and full-scale for blast pressures up to 25 psi with durations out to 100 ms full-scale. There is good potential for application beyond these limits as well, in particular for higher pressures. The ability of the panels/connectors to resist load is more greatly challenged by long durations than by high pressures.

1.0 Introduction

This paper reports on work completed by Wilfred Baker Engineering, Inc. (WBE) under contract to the Hoechst-Celanese Corporation (H-C) to develop structural upgrades for masonry control buildings common to their chemical plants. The author would like to thank Hoechst-Celanese for allowing publication of this information. These buildings are constructed with a relatively strong reinforced concrete frames but have minimally reinforced masonry walls and a light-weight concrete roof. The goal was to improve the blast resistance of the building. Particular attention was given toward developing upgrades to reduce hazards associated with wall failures. The focused attention toward walls was due to their construction, unreinforced masonry, which can serve as a debris source and due to the high magnitude of reflected wall blast loading. While a variety of wall upgrades were studied, only one is the topic of this paper, which is the use of pre-cast concrete panels and crushable shock absorbing connectors. Roof upgrades were also addressed in that project; however, this topic is not included in this paper.

Report Documentation Page

Form Approved
OMB No. 0704-0188

1. REPORT DATE **AUG 1996**	2. REPORT TYPE	3. DATES COVERED **00-00-1996 to 00-00-1996**

4. TITLE AND SUBTITLE **Blast Damage Mitigation Using Reinforced Concrete Panels and Energy Absorbing Connectors**	5a. CONTRACT NUMBER 5b. GRANT NUMBER 5c. PROGRAM ELEMENT NUMBER
6. AUTHOR(S)	5d. PROJECT NUMBER 5e. TASK NUMBER 5f. WORK UNIT NUMBER
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) **Wilfred Baker Engineering, Inc,8700 Crownhill Blvd,San Antonio,TX,78209-1128**	8. PERFORMING ORGANIZATION REPORT NUMBER
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)	10. SPONSOR/MONITOR’S ACRONYM(S) 11. SPONSOR/MONITOR’S REPORT NUMBER(S)

12. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution unlimited

13. SUPPLEMENTARY NOTES
See also ADM000767. Proceedings of the Twenty-Seventh DoD Explosives Safety Seminar Held in Las Vegas, NV on 22-26 August 1996.

14. ABSTRACT
see report

15. SUBJECT TERMS

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT **unclassified**	b. ABSTRACT **unclassified**	c. THIS PAGE **unclassified**	**Same as Report (SAR)**	**25**	

Standard Form 298 (Rev. 8-98)
Prescribed by ANSI Std Z39-18

The use of pre-cast panels placed on the exterior of the building satisfied one important design criteria. Being an exterior application, they minimize disruption to the operations being performed inside the building which controls process units. Because the panels are supported by the existing reinforced concrete frame, it was desired to avoid transferring loads in excess of the frame's capacity, thus eliminating the need to upgrade the structural capacity of the frame. This was achieved by the use of crushable connectors placed between the panels and the frame. A variety of connectors were evaluated during the project. The one that provided the best performance was a hexagon shaped steel tube. Figure 1 illustrates the wall panel and connectors mounted against an existing building and demonstrates the panel/connector concept.

The blast loading ranges of interest to the study were free-field pressures between 1 and 10 psi and shock durations between 20 and 100 ms. This range is representative of common chemical plant vapor cloud explosion scenarios. Walls oriented such that they receive a normal reflection will see an applied pressure of about 25 psi for a free-field pressure of 10 psi. The pre-cast panels and crushable connectors are capable of accommodating this range of interest; however, the worst case combination of 25 psi and 100 ms, while possible to achieve, results in construction difficulties for application of this upgrade method. The panel/connector system was more severely challenged by the relatively long duration loading than by the high pressure.

A combination of dynamic structural analysis and physical testing was used in order to evaluate performance of the pre-cast panels and the crushable connectors and to develop a design methodology. The testing was conducted at the WBE owned shock tube test facility which includes the capability to perform both small and large scale structural testing under blast loading conditions. A methodology was developed for determining both the pre-cast panel and connector structural requirements and dimensions. As will be discussed below, the two systems act together and hence cannot be analyzed independently.

2.0 Building Description and Pre-Upgrade Blast Performance

An example building was chosen for the study which is constructed with a substantial reinforced concrete column and beam frame system. It has masonry walls placed between the columns. The masonry walls include an interior layer of 4 inch concrete blocks and an exterior layer of 4 inch common brick, separated by a 2.75 inch air gap. The only reinforcement provided in these walls are horizontal wire trusses placed at every other course. The walls are connected to the columns by metal clips placed at the ends of each wire truss. These walls are capable of resisting conventional wind loads, principally through a one-way horizontal span and composite action between the two masonry layers. Lateral wind loads transferred to the columns are resisted by the reinforced concrete moment frames. In addition, some roof diaphragm and shear wall action can be expected. Columns are 12 x 12 inches with No. 8 bars positioned at the perimeter, three each face. Beams are similarly reinforced with a typical depth of 24 inches. Figures 2 and 3 provide an elevation and wall sections of the example building.

The minimally reinforced masonry walls provide some ductility and load resistance under conventional wind loads; however, they offer little resistance to blast loading. The walls will fail under low blast pressures as seen in testing, relative to those of interest to this study. This will be a

brittle mode of failure. For the study conducted, it was desired to upgrade the building to sustain pressures up to a 10 psi free-field blast load and durations out to 100 ms. This can result in reflected pressures as high as about 25 psi for a normal surface orientation. This load magnitude will cause wall failure with an attendant debris hazard. Even walls facing away from the source and experiencing a corresponding side-on loading will also fail. A demonstration test was conducted in the small WBE shock tube with a quarter-scale masonry wall. The wall was subjected to a full-scale load of 25 psi for 20 ms duration (25 psi for 5 ms at quarter-scale) with results as shown in Figure 4.

3.0 Development and Static Testing of Connectors

As indicated, it was desired to develop connections for placement between the panels and the frame which reduce the transfer of load to the frame components. The concept adopted utilized deformable steel connectors which crush under the blast load, thus cushioning the supporting building frame. A fall-out of this type of “soft” connector is that damage to the concrete panel itself is dramatically reduced; however, this was not the driving reason for developing the shock absorbing connectors. The concept is to provide connections which limit transferred load equal to or smaller than the static capacity of the supporting frame components.

Several different geometrical shapes were evaluated for use as connectors. These are illustrated in Figure 5. These various configurations were fabricated and placed under a hydraulic press in order to develop a load-deflection curve. The most dependable geometry was determined to be a hexagon shape. This geometry provides an elastic response under load until it yields when it responds plastically as the opposite faces are squeezed together. During the plastic response, if offers fairly constant load resistance. The elastic deflection is very small compared to the large plastic deflection. The plastic resistance does drop with deflection due to geometry-effects, but not greatly, resulting in a satisfactory load-deflection curve.

The load-deflection curve developed using the hydraulic press does not include the effects of high strain rates typical of quick structural responses to blast loading. High strain rates produce an increase in material yield and ultimate strengths. This effect has been well documented in literature [Baker, et al. (1983) and TM 5-1300 (1990), for example] and can be accounted for using textbook values of what are commonly referred to as dynamic increase factors (DIFs). The most pertinent of these values to the subject study are the low pressure or "far range" values in TM 5-1300 (1990) for ultimate material strength. The far range values are pertinent due to the relatively low blast pressures and long durations of interest to this study; that is, relative to military situations. The ultimate strength factors are more pertinent than the yield factors because the connectors undergo significant strain and hence primarily operate at strains well in excess of yield, thus, ultimate strength values are more appropriate. The DIFs reported for these conditions range from 1.1 for A36 (36 ksi yield) steel down to 1.0 for A514 (90 ksi yield) material. Thus, the DIF only have a minimal effect on connector strength, particularly for steels with higher yield strength than A36. During the blast testing that was conducted, a dynamic load cell was used to make measurements of loads transferred by the connectors. The dynamic load passed by the connector was measured to be approximately the same and often less than that measured in the static tests. Therefore, a DIF of 1.0 is recommended for dynamic analysis of these connectors.

The static capacity of the selected connector geometry can also be determined by analyzing the hexagon shock as a simple static frame component with ultimate strength properties. The hexagon shape results in a total of 6 plastic hinges which account for its ultimate resistance. The frame model of the hexagon shaped connector compares well with hydraulic press data for both the shock stiffness and its ultimate resistance. Such a model can be used to design hexagon connectors of arbitrary size.

Based on the results of the static press tests, a final connector configuration was selected for blast testing, the full-scale dimensions of which are presented in Figure 6. Figure 7 shows the quarter, half- and full-scale connectors that were used in blast testing. Included in the photos are specimens before and after they were subjected to static press tests.

The material used in fabricating the walls of the full-scale connectors was ASTM A-50 steel. This is a dependable, ductile material. Grade 50 steel provides for consistent strength requirements, whereas the strength of A-36 steel can vary between 36 ksi and 50 ksi (i.e., steel not meeting Grade 50 specifications often is stamped as A-36). Understated material properties will cause the shocks to be stronger than is specified, thereby increasing load transfer to the column or beam supporting the connectors.

The welds used in fabrication of the connectors were continuous and full strength. This insured that the connector plates deformed properly through the entire stroke and that the plastic hinges are formed to produce the full capacity of the connectors. The welds must not break during the deflection process. Connector test specimens should be manufactured and press-tested before implementation. This provides a load-deflection curve for the connector. Also, a specification sheet should be obtained for the material used to insure that the actual strength is known. Performing the press-test will allow for the quality of the welds to be checked, obtain the actual load-deflection curve for the connectors, and allow for the calculation of the resistance of the connectors. This will ensure that the connectors used in upgrading the buildings are of good quality and respond as expected during their deflection process.

4.0 Blast Testing

4.1 General

Physical testing was performed on quarter-scale, half scale and full scale test walls which were subjected to blast waves. Full-scale refers to dimensions which are relevant to the size of conventional buildings.

The blast tests were conducted in two shock tubes owned by WBE, a smaller one used for the quarter-scale testing and a larger one used for the half- and full-scale testing. Figures 8 and 9 include photographs of the small and large shock tubes, respectively. The shock tubes each consist of a driver and an expansion tube. The driver holds compressed air and is separated from the expansion tube by an aluminum diaphragm plate. The diaphragm plate is burst at a predetermined pressure and a shock or blast pulse is driven into the expansion section. The peak pressure of the blast pulse is controlled by the diaphragm burst pressure and its duration by the length of the driver. The expansion tube allows the blast wave to expand and achieve a geometry equal to the surface area of the test

article while maintaining uniform loading. The blast wave reflects off the test article without causing wrap-around back face loading or premature clearing relief. Blast pressure gauges are mounted along the edges of the target area to measure the applied load.

The quarter-scale testing was chosen based on the ability to conduct many tests with a quick turn around. This provided an adequate database to evaluate the proposed structural systems. Small-scale testing is a well developed engineering practice which demonstrates significant effects for dynamic loading and structural response. Baker, et al. (1973) provides an excellent overview of this practice. In order to generate pertinent data using small-scale testing, consistency must be maintained between important physical parameters in the tests. This is accomplished by maintaining ratios between parameters which have physical significance. Typically, these ratios are non-dimensional groups of terms which relate dimensions, strength, and time. The following non-dimensional ratios were identified in our analysis as having physical relevance and allowed comparison between small and large scale.

- The ratio of the shock wave duration to the natural period of the concrete panel (t_d/T_n)
- The ratio of maximum observed plastic deflection to elastic limit deflection. (x_m/x_e, also known as ductility ratio)
- The ratio of connector resistance to structural resistance of the panel (r_c/r_u)
- The ratio of connector stiffness to panel stiffness. (k_c/k)

The above ratios were used to relate similarity in responses at quarter-scale to that expected at full-scale. Later, the approach of organizing the test parameters into physically relevant groups will be discussed. This is a similitude modeling approach where the measured physical responses and test data can be organized and described by non-dimensional parameter groups, hence becoming independent of the size or scale of articles being tested.

Geometric scaling with similar materials was used in this test program. This type of scaling reduces all length parameters (spans, diameters, etc.) by the scale factor. While material properties such as yield stress, ultimate stress, compressive stress, Young's Modulus, and density are kept the same in model and full-scale materials. As spans were shortened and reinforcement bars diameters reduced using geometric scaling, thereby keeping the above described ratios were kept the same as the full-scale counterparts.

Time scaling was also maintained. Consider, for example, the natural period of an equivalent single-degree of freedom structure which is calculated by the established formula.

$$T_n = 2\,\pi\sqrt{\frac{m}{k}} = 2\,\pi\sqrt{\frac{\rho V}{k}}$$

where:

m = mass
r = resistance
ρ = density
V = Volume (length term cubed = ℓ^3)
k = stiffness

Material properties were kept the same in small and full-scale, by using similar materials, and can be combined with other constants. Also stiffness can be defined as proportional to

$$k \sim \frac{EI}{L^3}$$

Where E = Young's Modulus, which also is kept the same when using similar materials, hence becomes constant in this similitude analysis approach. Further,

L = characteristic span (a length term, ℓ)
I ~ wh^3, where w and h are characteristic width and thickness terms (length terms, ℓ)

thus,

$$k \sim w\,\frac{h^3}{L^3} \sim \frac{\ell^4}{\ell^3} \sim \ell$$

and

$$T_n \sim \sqrt{\frac{\ell^3}{\ell}} \sim \ell$$

Thus, when geometric scaling with similar materials are utilized, the natural period varies proportionally to the length term and hence must be reduced by the scale factor. In order to keep the ratio t_d/T_n constant between full and small-scale, the load duration t_d must also be reduced by the scale factor.

The above defined ratios also dictate that peak applied blast pressure remains the same at small- and full-scale. This is because unit slab resistance is unchanged when geometric scaling is used and similar materials are employed. Thus, the ratio r_u/p dictates that the blast pressure remains the same

at small-scale. Table 5 shows a dimensional comparison between the quarter-scale, half-scale, and full-scale parameters.

Table 1. Dimensional Comparison between Various Scales Used in Testing.

Parameter	Full-Scale	Quarter-Scale Model	Half Scale Model
Reflected Blast Pressure	p_r	p_r	p_r
Blast Duration	t_d	$(1/4)*t_d$	$(½)*t_d$
Reflected Blast Impulse	i_r	$(1/4)*i_r$	$(½)*i_r$
Slab Resistance	r_u	r_u	r_u
Spans and Thickness	L_i	$(1/4)*L_i$	$(½)*L_i$
Reinforcement Area	A_s	$(1/16)*A_s$	$(1/4)*A_s$
Connector Dimensions and Wall Thicknesses	L_i	$(1/4)*L_i$	$(½)*L_i$

4.2 Testing

The quarter-scale tests included hexagon connectors attached to concrete panels and tested using the small WBE shock tube. The connectors were tested over the blast loading range of interest, including pressures up to 25 psi peak reflected and durations ranging from 5-25 ms small scale (20-100 ms large scale). A total of eight quarter-scale tests involving panels and connectors were conducted.

For large-scale tests, half- and full-scale connectors were fabricated and placed on 8 ft x 8 ft reinforced concrete panels. The half-scale tests modeled 16 ft x 16 ft spans. Four full-scale and one half-scale tests were conducted. The full-scale shocks mounted to a concrete panel, before and after testing, can be seen in Figure 10.

In the large-scale tests, a dynamic load cell was placed between the shocks and the test frame. This was done to confirm the assumption that the frame would not experience a load greater than the shock capacity. The peak measured loads were equal to or less than the static load test measurements. The large-scale tests also demonstrated that the panel/connector concept was easy to field implement. Attaching the connectors to the concrete panels and mounting the panel to the test frame posed no particular difficulties. Figure 11 includes a photo taken of a large panel with connectors being loaded in the shock tube.

Two of the full-scale tests included a masonry wall placed behind the concrete wall panel in order to simulate the masonry of the example building. The masonry did fail due to the internal pressure developed between the concrete panel and the walls which was measured during the testing. The internal pressure could be explained by the simple relationship, $P_1V_1 = P_2V_2$. Here the subscript 1

indicates pre-test pressure and volume in the cavity while the subscript 2 pertains to conditions at maximum deflection of the panel/connector system. The nature of the masonry wall collapse when the panels/connectors are used is different from that which was seen in the tests where the masonry wall was directly loaded with the blast wave. For the former, the masonry wall fails and produces high speed projectile debris. For the latter, the masonry wall was loaded by the internal cavity pressure which is relieved with the wall failure and the wall tended to fall over, or tossed, without producing the high velocity projectile debris; however, the result is still considered hazardous. Thus, it is recommended to either remove the masonry walls or provide a window-sized opening to allow venting of the air space between the walls. This will require weather-proofing the pre-cast panel walls. This is an interesting reversal of the trend to recommend "bricking-in" windows of control rooms.

5.0 Wall Upgrade Design Methodology

The testing provided good insight into the performance of connector supported panels and generated a substantial database. To complete the study, it was necessary to select a generalized prediction model which can serve as a design procedure and which is applicable over the entire range of interest.

The panel and connectors were analytically modeled as a simple rigid-plastic system, whose resistance was provided by the shock and whose mass is provided by the panel. Reference Baker, et al. (1983) for a detailed discussion of this approach, which was taken for two reasons. First, the testing demonstrated minimal or no permanent panel deformations while the connector deformations were large. Thus, it was reasonable to decouple their response and model the response as a single-degree-of-freedom (SDOF) system. For this to remain true, the connector resistance must be lower than that of the panel. Based on the testing, when the connector resistance is at most half the panel resistance, the connector yields first and the panel is minimally damaged. The system was idealized as all deflection occurring in the connector while mass is provided by the panel and is 100% effective; i.e., a mass factor of one. Second, the elastic portion of the connector response can be neglected since it is much smaller than the plastic response, hence mimicking a true rigid plastic response.

The rigid-plastic model and its governing equations are illustrated in Figure 12. Comparisons of predictions with test measurements indicated that the SDOF rigid-plastic model resulted in conservative overestimates of the permanent deformation of the connectors. Figure 13 provides a comparison between the rigid-plastic analytical model and the test data using a pressure-impulse diagram format. The test data includes that generated at all three scales. The asymptotes in this figure were developed by established principles as follows.

The pressure asymptote is achieved by equating work with strain energy as follows:

$$Work \; = \; p * X_m$$

$$Strain \;\; Energy \; = \; r_c * X_m$$

equating gives,

$$p * X_m = r_c * X_m$$

or,

$$\frac{p}{r_c} = 1.0$$

(Where p = pressure, X_m = maximum deflection, and r_c = connector resistance.)

The above ratio is a scaled pressure term relating pressure to connector resistance. Since both terms have the same units, the ratio is unitless.

The impulsive asymptote is achieved by equating kinetic energy with strain energy and taking advantage of the impulse-momentum relationship where velocity equals impulse divided by mass.

The impulse-momentum relationship is simply a rearrangement of the commonly used expression F = ma as follows:

$$F = Ma$$

$$F = M\frac{dv}{dt}$$

$$F\ dt = M\ dv$$

$$\int F\ dt = \int M\ dv$$

$$I = M(v_i - v_f)$$

$$where\ v_i = 0, \quad dividing\ by\ loaded\ area\ gives$$

$$i = mv \quad (unit\ mass\ ,\ unit\ impulse)$$

$$finally, \quad v = i/m$$

The above was simply put by Newton when he expressed his second law by stating that the rate of change of an object's momentum is in proportion to the net force acting on that object.

Equating kinetic energy and strain energy gives,

$$Kinetic\ Energy = \frac{1}{2}\ m\ v^2 = \frac{1}{2}\ m\ (\frac{i}{m})^2 = \frac{i^2}{2m}$$

$$Strain\ Energy = r_c * X_m$$

equating gives,

$$\frac{i^2}{2m} = r_c * X_m$$

or,

$$\frac{i^2}{m\ r_c\ X_m} = 2$$

(Where m = mass, *v* = velocity, t = time, F = force, X_m = maximum deflection, i = specific impulse, and r_c = connector resistance.)

The above ratio is a scaled impulse term, which is also non-dimensional. The theoretical curve between the two asymptotes was developed using numerical integration of the basic equation of motion in Figure 12. Note that the P-i diagram with the scaled pressure and scaled impulse terms are independent of physical scale (full scale, half scale, quarter scale, etc.) since non-dimensional terms are utilized.

A review of Figure 13 indicates that the bulk of the test data falls above and to the right of the P-i diagram. For the tests, the "known" values were r_c, m, i, and p and the measured response was X_m. Thus, the P-i curve tends to over predict X_m, since X_m is in the denominator of the scaled impulse term. Thus, using the theorectical curve results in a conservative prediction methods.

The curve in Figure 13 was fitted to an equation which relates the scaled impulse and pressure term over the entire range of values. That curve fit equation is as follows.

$$\frac{i^2}{X_m\ m\ r_c} = \frac{7.6}{(\frac{p}{r_s} - 1)^{1.8} + 2}$$

This equation can be used to size the panel based on inputs of blast pressure, impulse, and connector stroke. This leaves connector resistance and panel mass. Typically, the connector resistance will be limited by the supporting component such as a column or beam and the mass is then calculated.

6.0 Design Example

The above relationship can be used to design the panel/connector system. The design approach basically takes the following steps:

1. Determine the capacity of the weakest supporting element such as the column, beam, roof diaphragm or foundation. Set the connector load capacity on the face of the panel supported on this element to this value or less. Set the strength of the connectors on all other faces equal to this value in order to create a balanced support for the panel. Thus, a total load capacity of the connectors are established.
2. Calculate the total length of the connectors to be placed about the perimeter of the panel by comparing with the press data for the connector .
3. Calculate the unit resistance of the connector by dividing the total load capacity of the connectors by the panel area.
4. Using Figure 13 or the equation above, calculate the required panel mass. Known values will be pressure, impulse, connector unit resistance, and the available connector stroke. This allows for a direct calculation of panel mass which relates to panel thickness.
5. The panel reinforcement is established by providing adequate strength to the panel such that it can resist the blast loads and transfer that load to the connectors. This is accomplished by designing the panel to resist the blast loading under the assumption that the panel is rigidly supported. Methods in TM 5-1300 can be followed to size the reinforcement and to check shear.
6. Typically, some iteration is necessary to achieve a balance between the panel thickness, and hence its mass, and the load capacity of the connectors. The panel thickness can be reduced, but this requires an increase in load capacity of the connectors, which can compromise the supporting frame. The stroke of the connectors can be increased by constructing deeper hexagon sections. A review of Figure 13 indicates that increased connector stroke allows for decreased panel thickness and/or connector capacity; however, a new connector geometry must be analyzed or tested to establish its resistance-deflection properties.

This approach was taken to size connectors and the panel for the example building with panels spanning 14 ft X 14 ft with connectors as illustrated in Figure 6. The connector has an average resistance of 580 lbs/inch of hexagon tubing length. This example assumes that the panel is supported by the column, roof diaphragm and the foundation as shown in Figure 1. The weak link on this system is the column which has a static line load capacity of 840 lbs/inch. Columns must support panels on each side.

For an applied pressure of 25 psi and a triangular load duration of 40 ms, it was determined that a 17 inch thick panel supported by a total of 28 ft of connector length would protect the building. This provides for 7 ft of connector length per side, using the tested connector shown in Figure 6. The seven feet of connector can be provided by seven one foot increments spread evenly along the side. See Figure 10 as an example. With this configuration and each column supporting two panels, a column is loaded by a total of 14 ft of connector, thus continuous over the column.

7.0 Closure.

WBE has completed development of an upgrade methodology applicable to industrial control buildings. The upgrade entails the using energy absorbing connectors placed between a pre-cast reinforced concrete panel and the existing building frame. The energy absorbing supports are hexagonal pipe sections which will absorb the blast energy during plastic deformation. The approaches taken to upgrade the control building makes use of the existing frame capacity. This eliminates retrofit of the building's reinforced concrete frame.

The panels/connectors were developed for blast loads common to industry hazards and limited for this project as pressures up to 25 psi and durations up to 100 ms. The latter, load duration, when taken to large values, poses greater difficulties for the panel/connector system than does pressure taken to large values. When the load duration is very long, a deep connector with a large stroke is required before bottoming out. Thus, long duration loads in combination with constructibility constraints limit the effectiveness of the connectors. Constraints on pressure, however, were not evident. It is anticipated that, for uniform loads, the panel/connector system should be effective for very high pressures with relatively short duration loads and corresponding impulses. The system is very effective for impulsive loads and should therefore have military applications.

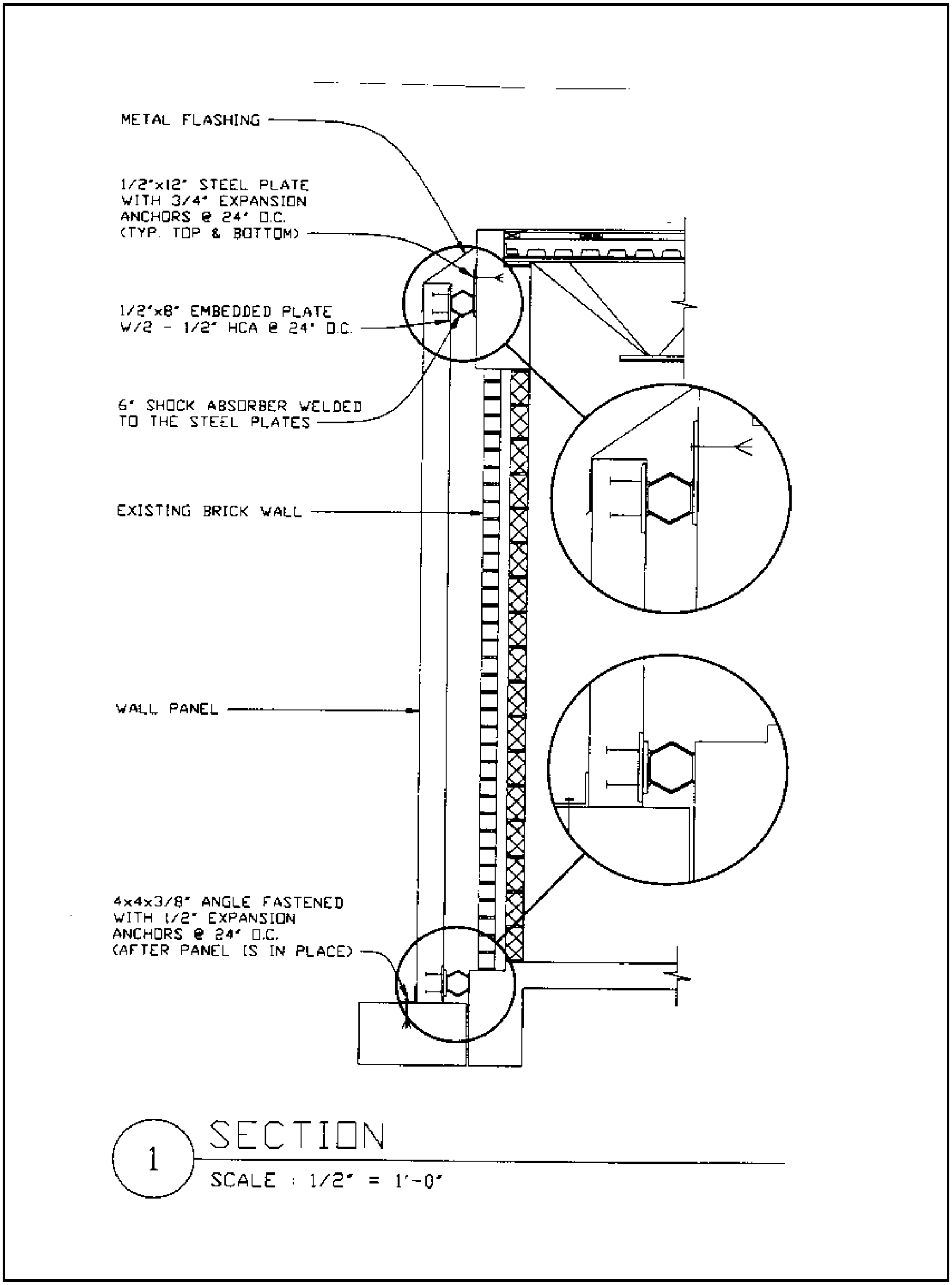


Figure 1. Schematic of Panel With Connectors In-Place on a Building

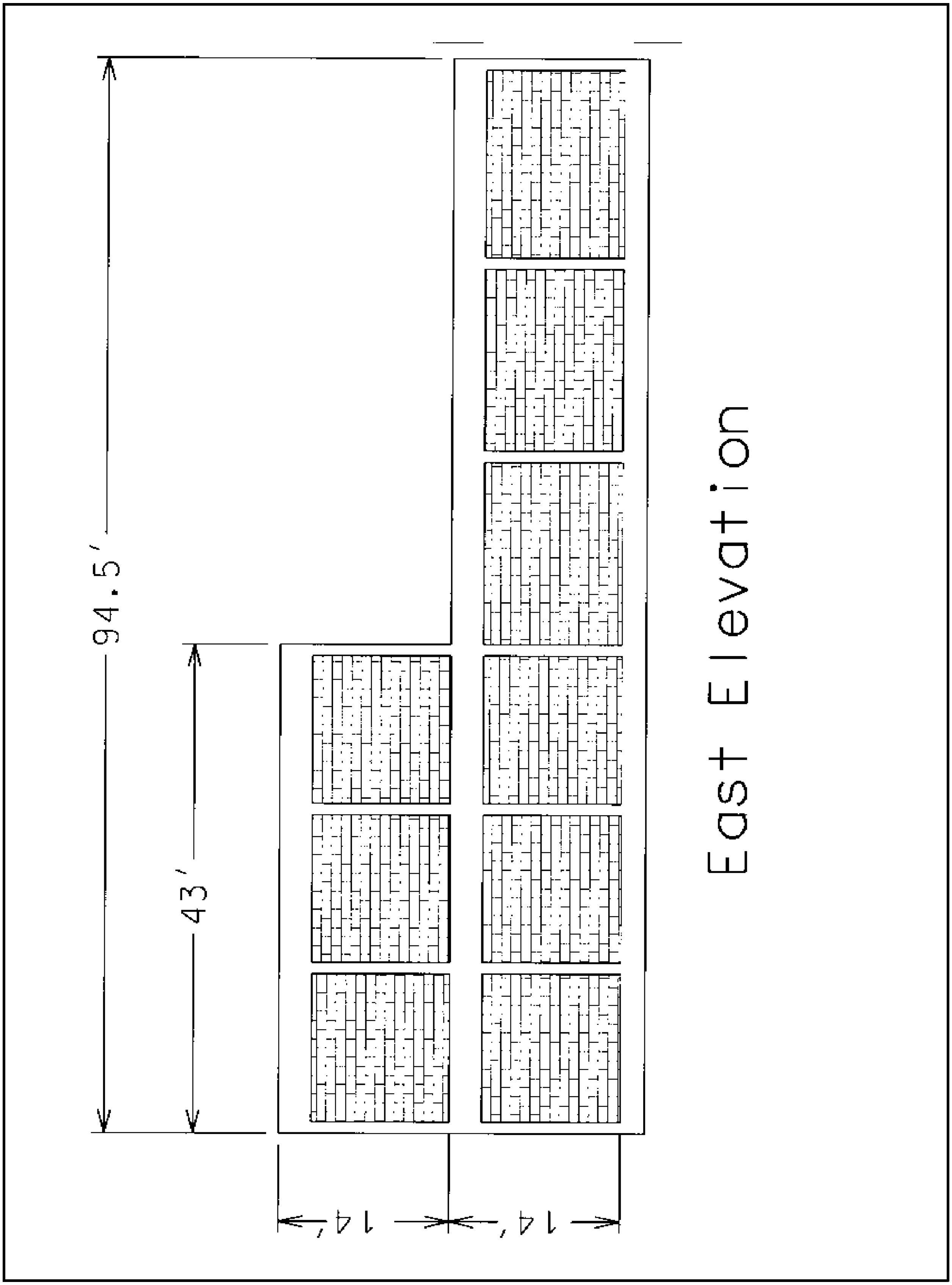


Figure 2. Example Building Elevation

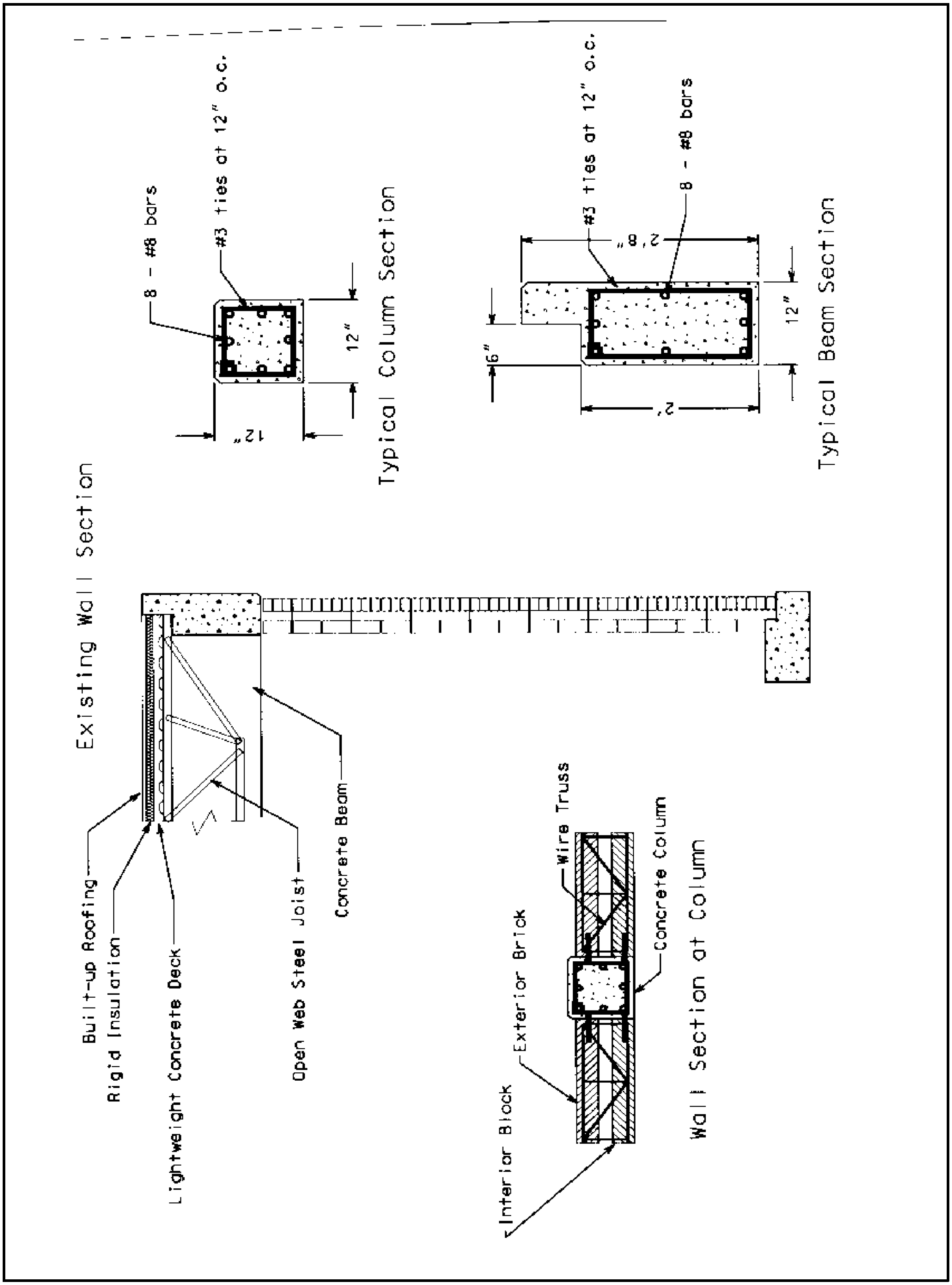


Figure 3. Example Building Sections

Figure 4. Debris From a Quarter-Scale Unreinforced Masonry Wall

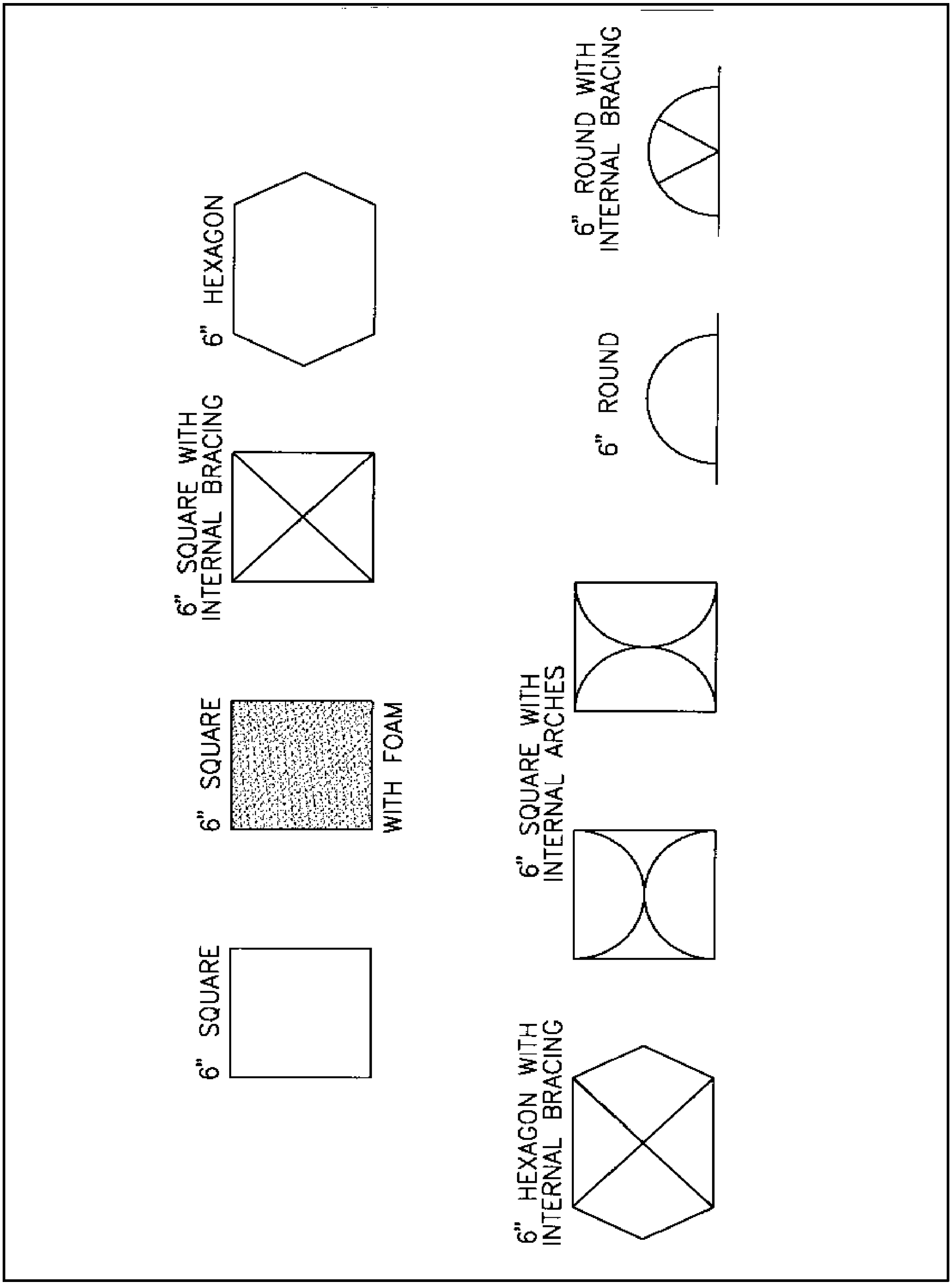


Figure 5. Various Configurations Evaluated for Use as Connectors

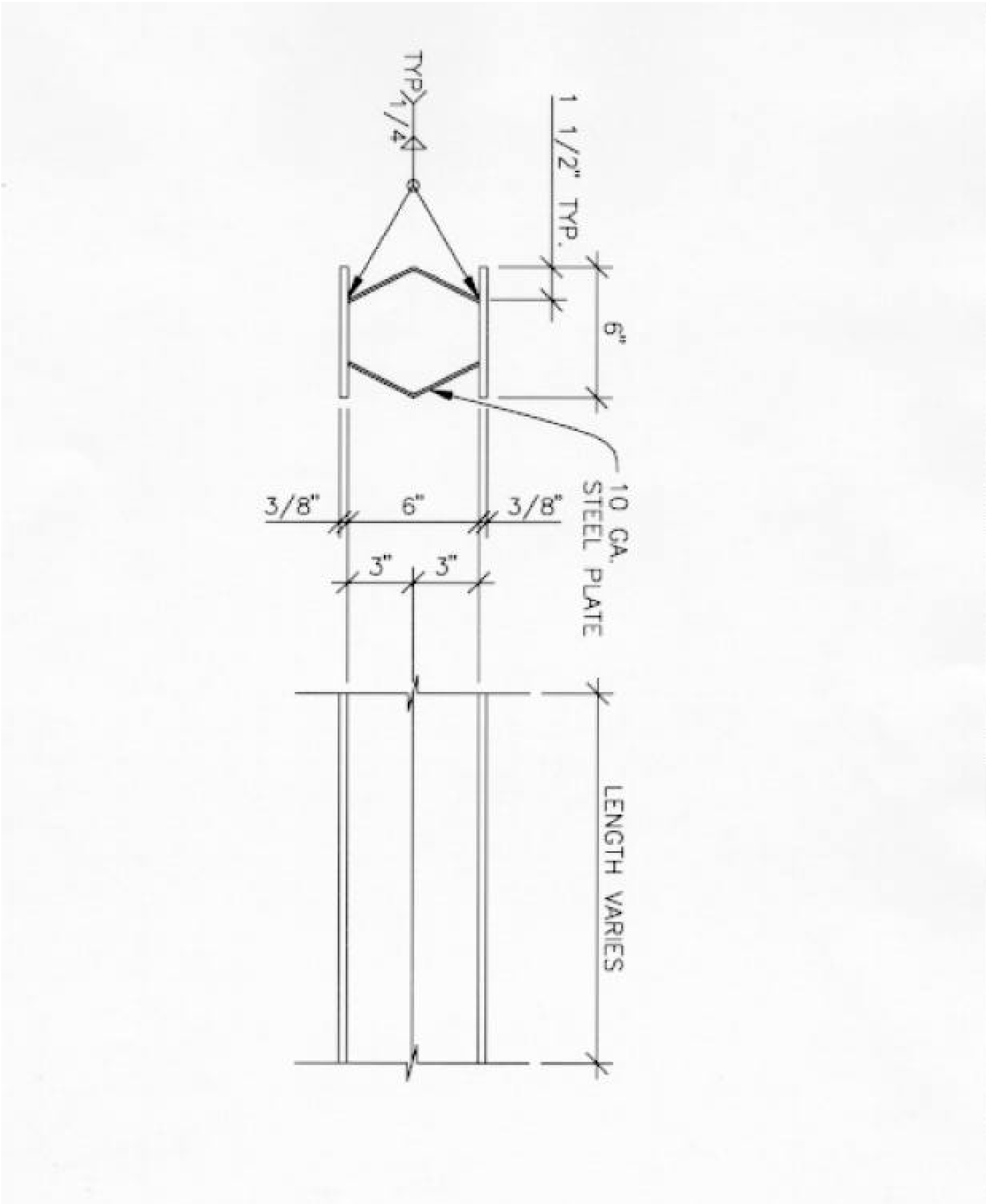


Figure 6. Connector Selected for Testing - Full Scale Dimensions

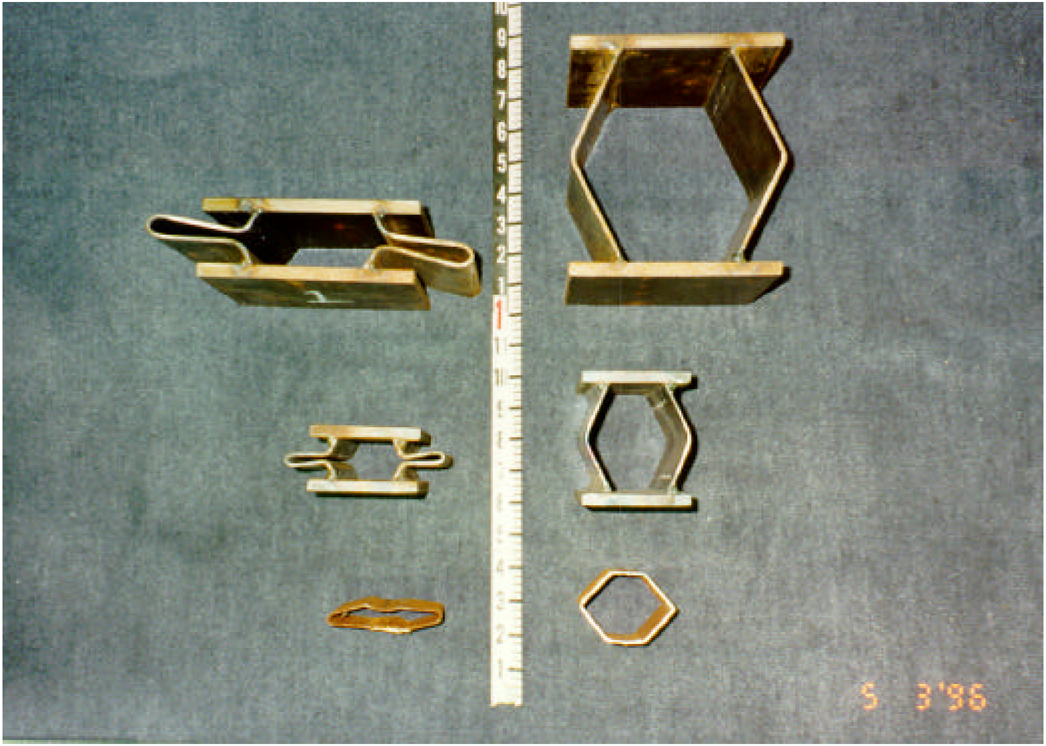


Figure 7. Full-, Half, and Quarter-Scale Connectors, Before and After Pressing

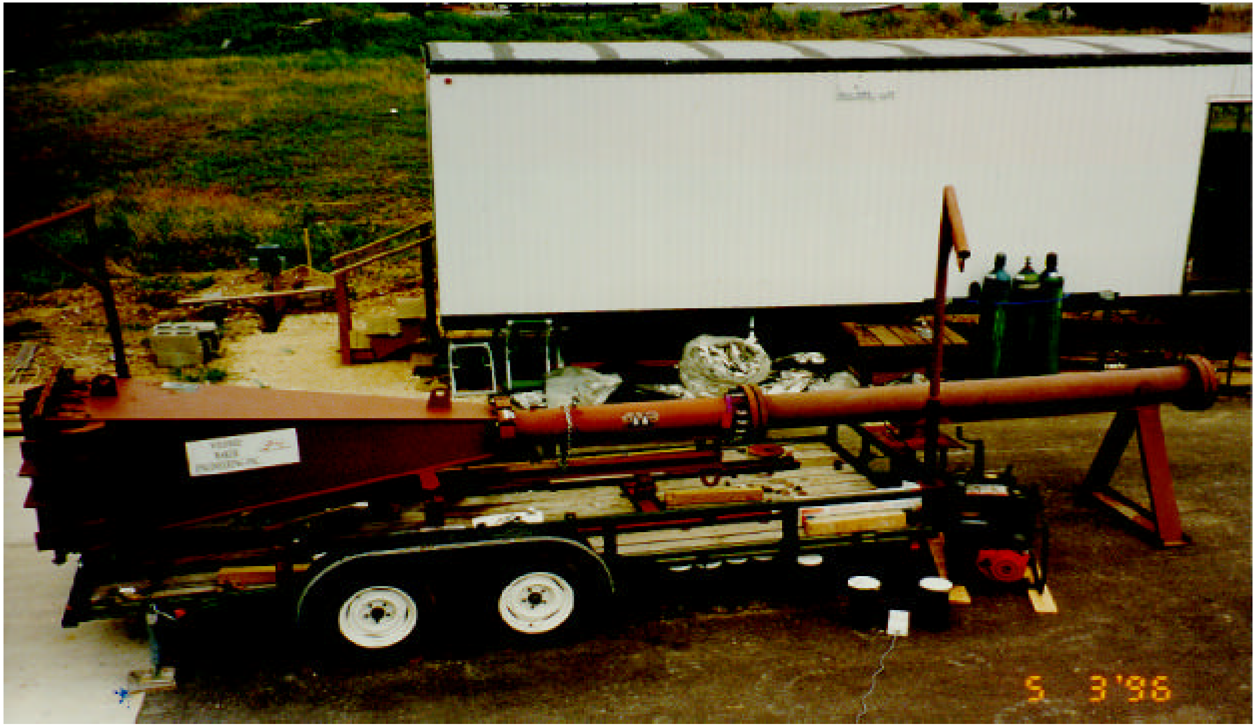
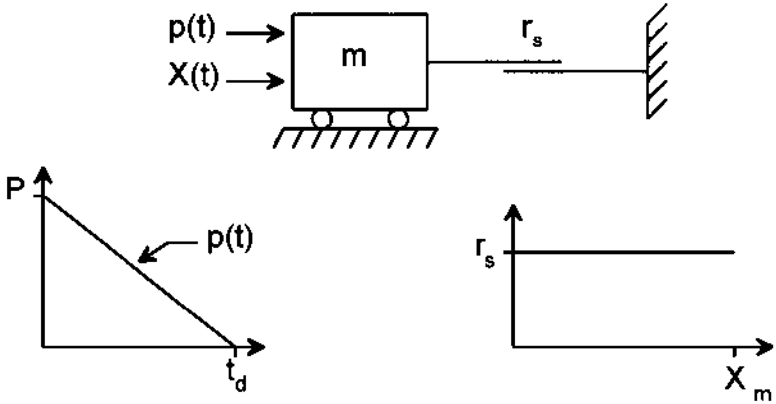


Figure 8. Small Shock Tube

Figure 9. Large Shock Tube

Figure 10. Connectors Mounted on Panels, Before and After Testing

Figure 11. Panels Being Loaded into Shock Tube Test Frame



$$\text{rigid-plastic:} \quad p(t) - r_s = m\ddot{x}$$

Rigid - Plastic SDOF System

r_s = shock resistance
m = panel mass
p(t) = blast pressure history
X_m = maximum deflection
X(t) = deflection history

Figure 12. Rigid-Plastic SDOF Model

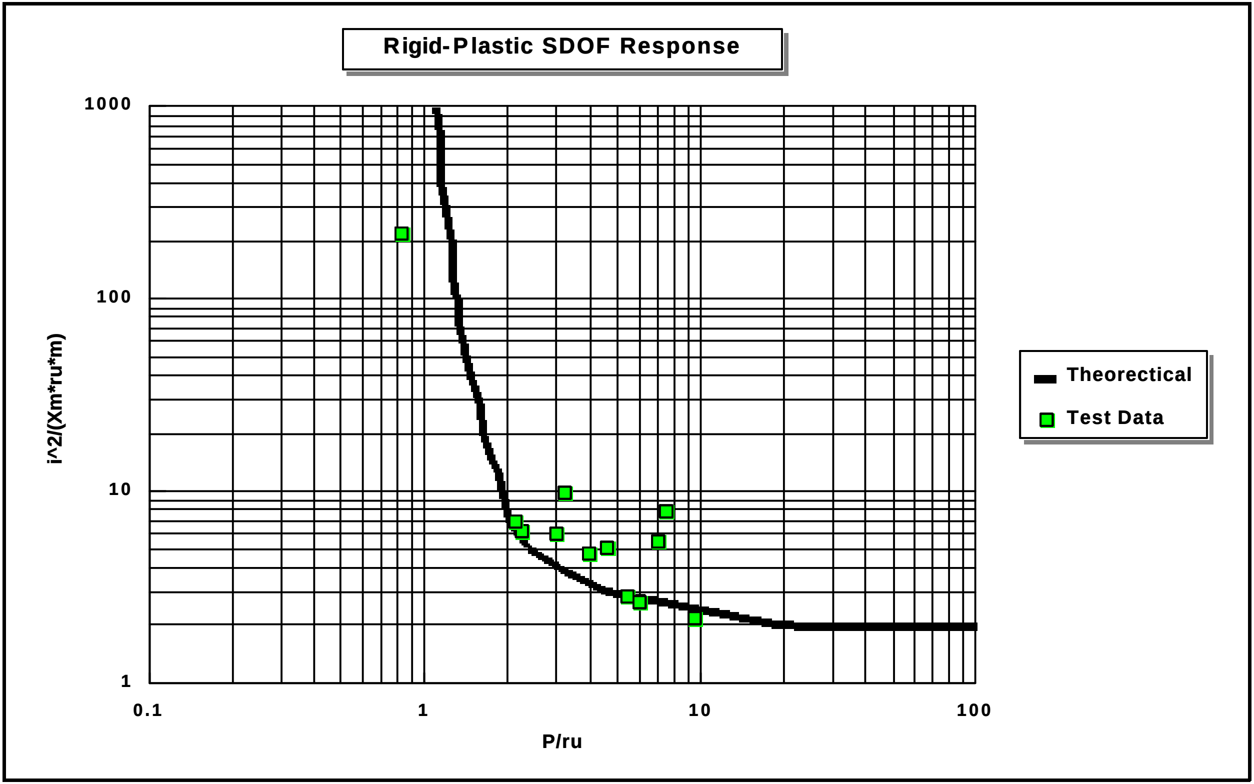


Figure 13. Predicted Non-Dimensional Pressure-Impulse Curve Compared With Test Data